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SHRINKAGE OF CONCRETE IN ACTUAL STRUCTURES

BY WILLIAM M. BASSETT *

(Presented at a meeting of the Designers Section of the Boston Society of Civil Engineers
on December 10, 1930)

FROM a study of small sized concrete members in laboratories it is known that concrete shrinks upon loss of free moisture, either by chemical combination or by evaporation; that it also shrinks upon loss of internal heat; that it expands upon the absorption of moisture and varies its dimensions corresponding to changes in external temperatures. From these laboratory tests it has been determined that the ultimate longitudinal shrinkage usually amounts to about .055 per cent when opportunity for complete loss of all uncombined water and all chemical heat presents itself; also that as the loss of this uncombined water is retarded so is the rate of shrinkage, and further, that when completely wrapped in wet burlap so as to maintain a condition of continued 100 per cent humidity the shrinkage is nil and actual expansion in small amounts may result.

Full sized structures of concrete, as revealed by the opening of construction cracks and development of intermediate cracks, testify to the shrinkage of concrete. These structures also show that while it is possible, by reinforcing, to force the intermediate cracks to definite locations, they can rarely be eliminated. In hydraulic work a designed or anticipated crack is no menace to a good structure if it is properly built, but an intermediate crack that just happens may admit water and frost to the ultimate deterioration of the structure.

The New England Power Construction Company and its consulting engineers have endeavored to design and build the large concrete struc-

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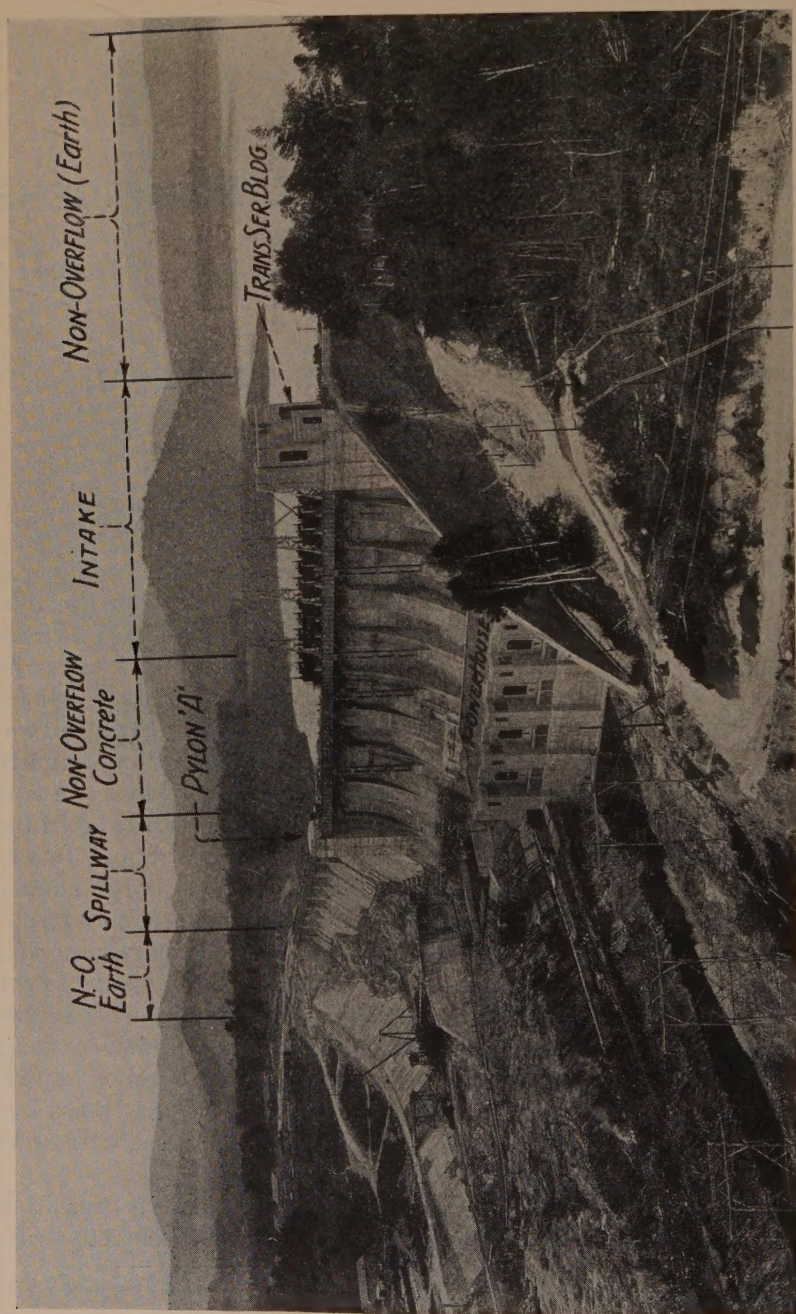


FIG. 1. — ARRANGEMENT OF PRINCIPAL STRUCTURES, 15-MILE FALLS, LOWER DEVELOPMENT

ures of the Fifteen Mile Falls Lower Development so that changes in bulk might work minimum damage to the structure. It is the intention in this article to select a few of the structures which will demonstrate this matter to best advantage and indicate where the construction joints were located and why, and finally to let the structures themselves testify as to what has taken place.

The general arrangement of the structures which comprise the dam at the lower Fifteen Mile Falls site on the Connecticut River is described below. Fig. 1 shows the power house, and immediately upstream from that is the intake. To the left is the concrete non-overflow part of the dam, then the 850-foot spillway, and beyond is the west earth dike. To the right of the intake is the transformer service building and freight shaft, and then the east earth dike. Downstream from the spillway is the discharge channel with its training wall (located just below Pylon A) on its right, and with the paving on its left.

SPILLWAY STRUCTURE

We will now investigate the spillway, relating how it was planned, built, and how, to date, it has worked as a structure. For the economic use of the topography the spillway was so located that it interposed itself between the railroad on the Vermont shore (at the left-hand side of Fig. 1) and the power house. The railroad provided the means of bringing to the plant heavy equipment, practically all of which is located in the power house group, and this railroad also, throughout the operating life of the plant, will permit shipments of parts in or out. The bridge on the dam for this railroad was designed for the economic span length, which worked out to be about 77 feet. The bridge is made up of plate girders supported on concrete piers. The contractor wished to build one face of each pier as a continuation of a spillway bulkhead which established 77 feet, or a portion of it, as the length of a spillway section.

SHRINKAGE CONSIDERATIONS

A study was then made of these 77-foot sections of spillway as to shrinkage. It is known that concrete, when completely cured, dried out and not restrained, shrinks about .055 per cent of its length, which for a 77-foot section would be .5 inch. Therefore if it is considered that the upper 2-foot layer of the spillway is separated from the portion below by a greased plane, it should ultimately lose .5 inch of its length, and the construction joints would become $\frac{1}{2}$ an inch wide. The lowest 2-foot layer in contact with the ledge, and restrained by it, would probably

shrink a very small amount. The next 2-foot layer would have a tendency to effect some shrinkage, but would be restrained by the lower 2-foot layer almost as effectively as the latter would be by the ledge. It is apparent that in successive layers, farther away from the ledge, the restraining effect would diminish, and it would be found that the top layer would acquire some proportion of its theoretical .5 inch of shrinkage. Considering this from another angle, it was figured that the upper layer could take a tension of 250 pounds per square inch, or a stretch of .09 inch, without cracking, assuming $E=2,500,000$ pounds per square inch. A bulkhead crack of .41 inch would absolutely eliminate any intermediate cracks. A curving of the bulkhead plane of .20 inch in 40 feet or more did not seem impossible. There was a fair chance of no cracks in the top layers, but a certainty that intermediate cracks would appear somewhere in between the top and the bottom.

CONSIDERATION OF POURING FOR MONOLITHIC CONSTRUCTION

The next consideration of the 77-foot section was from the standpoint of pouring for monolithic construction. The plant could produce about 100 cubic yards per hour with its four 1-cubic yard hoppers, provided (1) that the concrete cars could get away quickly enough, (2) that the buckets could get prompt enough derrick service at the ends of their runs, and (3) that they could be poured immediately and returned. The terminal facilities at the ends of the car runs usually cut this down to not over 50 cubic yards per hour for any one point. The 77-foot sections near the bottom of the spillway, where the base is at least 60 feet through, produced 4,620 square feet of area, and for a spread of 18 inches in thickness there would be required 260 cubic yards per spread. This required, therefore, at least five hours between coverings, which was too long a time.

As a matter of interest, in a few cases later, when placing the big east retaining wall, 700 cubic yard pours were made and 4,500 square feet were covered at a single pour, but an inspection of these operations indicated clearly that the plant was operating near the limit.

In addition, in order to assure the completion of the spillway bridge as soon as possible, the piers were placed upon short sections of small yardage. From this point of view also, it became desirable to break up the 77-foot lengths. The solution became dependent, however, not on the question of how short could the short sections be built, but how long could the long sections be built.

With 45-foot sections the theoretical unrestrained shrinkage would be

30 inch, and with .054 inch deducted for tension there would be left .246 inch to be taken up at the top by bulkhead joints, if it were certain that there would be no intermediate cracks. The curving of .123 inch for each joint face at the general height of about 40 feet appeared reasonable. Near the bottom, where restraint from the rock was expected, there would still be a chance of some intermediate cracks, since it was fairly certain that the bulkhead joint would not open up .246 inch near the edge. However, by that time there had been poured a portion of the east retaining wall with sections 40 to 45 feet long with no intermediate cracks. Furthermore, there was small chance that the intermediate cracks, if there were any, would be able to run through to the upstream face, since they would have to pass through about 32 horizontal feet, even when only 25 feet from the top of the spillway. In traversing this distance such cracks would have to pass through a concrete core, retaining practically all of its chemical heat and original uncombined moisture, and long before these would have suffered any reductions the pond would be up, and the upstream face would be subject to continuous saturation. Any intermediate cracks, therefore, had little chance of becoming "wet cracks."

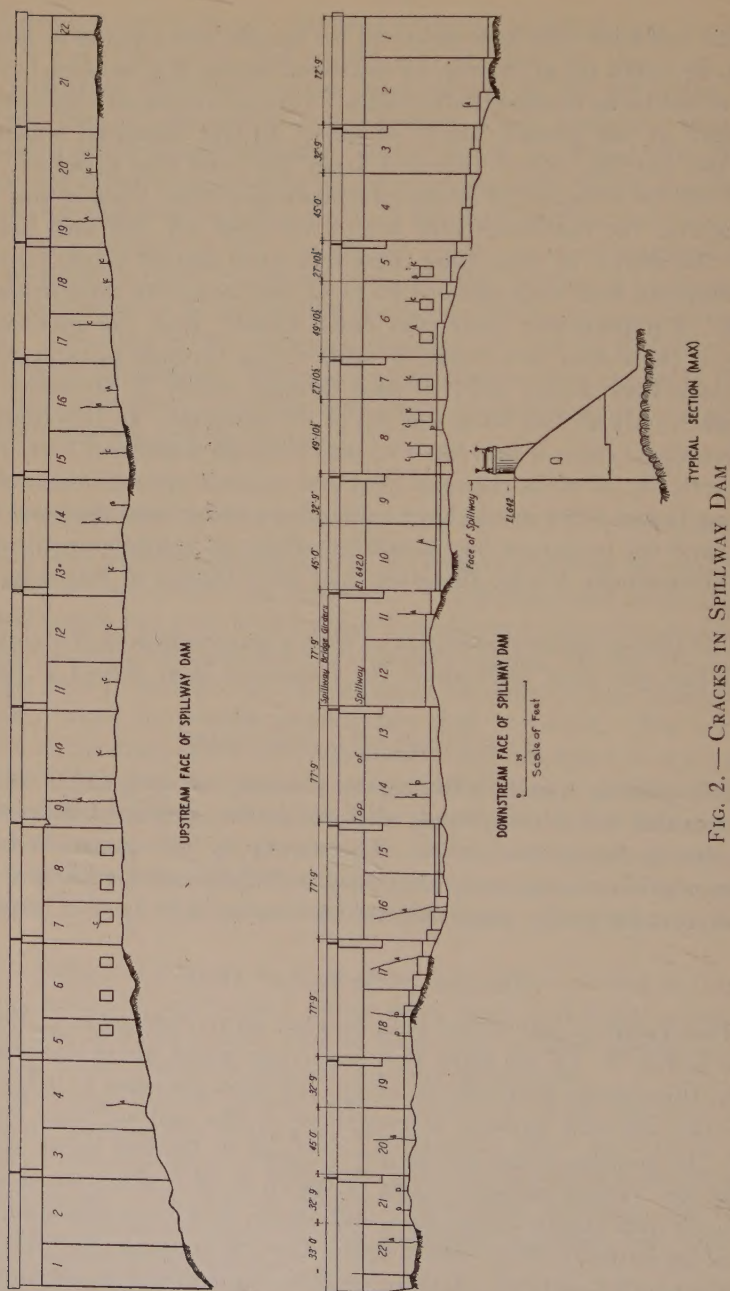
The 45-foot sections worked out with a maximum area of about 2,700 square feet per pour, well under the maximum later placed at the east wall.

THE COMPLETED STRUCTURE

The spillway was built during the summer and early fall of 1929 and by November it was completed. It stood in the open until the pond was filled during September, 1930. The survey of the upstream face, of course, continued only up to that date, and these comments refer to the downstream face only, since this will be continuously open to inspection.

CRACKS IN SPILLWAY DAM

The results of our actual field surveys up to November 1, 1930, are shown in Fig. 2. Of the eight 45-foot sections which are of uniform cross section throughout each length, except for variations due to ledge elevations, two are still without intermediate cracks and six have acquired them. In general, these cracks are confined to the less bulky sections. Of the ten 32-foot sections, each of which is of uniform cross section within its own length except as noted above, four have acquired cracks and six are without them. Here, again, the cracks are in general confined to the less bulky sections. It appears from the decrease in the number of cracks, in going from the 45-foot to the 32-foot sections, that a length of



something under 30 feet would have produced a spillway free from intermediate cracks.

A further study of this diagram shows that the intermediate cracks all start from the ledge and disappear before they reach the top. Due to the restraining effect of the ledge it would be expected that the bulkhead joints would be wider at the top than at the bottom, and the field measurements indicate this to be a fact.

There are twenty-three bulkhead joints, and their total openings at their tops, to date, amount to 52.32 inch, or .016 per cent, as against .055 per cent, which is the theoretical unrestrained and totally dried out shrinkage. At the top of the spillway toe bench there was a total of 27/32 inch, or .008 per cent, and at the ledge itself there was 17/32 inch, or .005 per cent. Many of these intermediate cracks run to ledge. The rock, therefore, acts just as if the base had been tied with heavy reinforcement. Where it is restrained there are several thin field cracks, and where unrestrained a few large bulkhead ones. These intermediate cracks have not yet penetrated into the spillway far enough to show in the tunnel side walls.

The diagram also shows intermediate cracks starting off from each corner of each low-pressure sluice opening. This is what would be expected, since cracks practically always appear where the concrete suffers an abrupt change in section. Furthermore, there were noted, also, here and there in the tunnel roof, longitudinal cracks, indicating how an abrupt change in section affects a concrete structure.

The field figures indicate that in general the bulkhead joints at their tops are .071 inch wide. With a theoretical unrestrained ultimate shrinkage of about .30 inch, there is therefore a stretch of about .229 inch. The tension without cracking might take up .054 inch of this, but since the cracks extend only part way up the face of the spillway, the tension distortion may not amount to more than .04 inch; the other .189 inch is due in part probably to the continuous restraining effect of the warmer and more moist layers below, and also to the fact that the top portion itself has not reached the condition of ultimate shrinkage. Unfortunately there are no accurate measurements of joint openings as of last winter, but in general it is known that the joints have closed somewhat since the pond was filled. Part of the .189 inch is doubtless due to the pond. Very little of this can be charged to the alternate pouring of sections, since in some cases adjacent sections were hardly more than one pour ahead of each other.

It is interesting to note that one bulkhead joint, which late last winter showed a quarter-inch opening at its top, has, since the filling of

the pond, decreased to about $1/16$ inch. This, of course, is in part because of the warmer weather during the summer, but it is believed that it is also due to the expansion of the concrete in the upstream face of the spillway, owing to saturation.

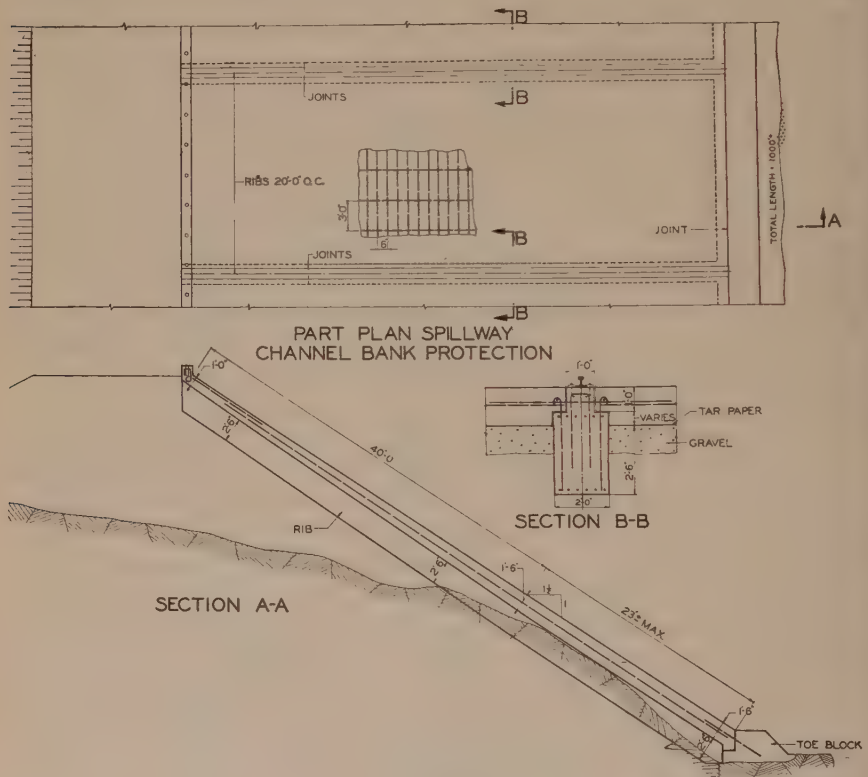


FIG. 3.—SPILLWAY CHANNEL BANK PROTECTION

SPILLWAY CHANNEL SLAB

Another problem was the protection of the earth bank of the spillway channel which will handle the flood waters after they have passed over the spillway. This protection took the form of a slab, the general construction of which is shown in Fig. 3. This slab foots on a toe block which is built on good, rough ledge throughout its length. This toe block was built first; and later, extending up the bank, there were built concrete ribs on about 20-foot centers with shoulders near the upper face for carrying the paving slab. This upper face was built flush with the exposed face

of the slab. By this means the slab was divided into convenient lengths of about 19 feet, and this scheme also provided a bed for tracks upon which to advance the forms up the sides of the banks. The permanence of the construction depends almost entirely upon the complete absence of any irregularities in the surface of the slab, since during flood velocities would be encountered as high as 50 feet per second. It was desired to keep this structure as devoid of cracks as possible. It was not expected, however, that it was possible entirely to eliminate them, and therefore the slab was designed so as to confine cracks to the portion of the slab supported by the ribs.

The slab was provided with a center reinforcing system of .5%. This system was carried through the planes between the faces of the rib top and the slab ends. The ribs themselves were reinforced longitudinally with upper and lower systems, the total per cent running about 2.6. These were built partly on original earth and partly on fill which had gone through a winter, spring and summer. The slabs were built upon fill which was rolled and carefully tamped. The slabs as reinforced are capable of spanning from rib to rib, together with either a certain amount of pressure from the water outside or from the bank behind.

The slabs, of course, are fairly thin, and have probably lost all their curing heat and uncombined moisture. There is a crack between each slab and its rib, and these cracks vary in thickness from a hair line to about $\frac{1}{16}$ inch. At any intermediate cross section there is in tension, acting against shrinkage, for every square foot of slab cross section 143.3 square inches of concrete section plus 8.6 square inches of steel (the latter rated as concrete equivalent), making 151.9 square inches of concrete tension cross section. In the plane, between the side of the rib top and the slab end, there is only the steel for tension capacity, namely, 8.6 square inches (concrete equivalent). With such a contrast in section the shrinkage cracks, if any, would come where desired, namely, where supported by the rib and along the face of its head.

WHY CRACKS APPEARED IN SLAB

Having designed the slab so that cracks, if any, would occur in the slab ends, the next problem to appear was, "Why should there be any cracks at all?" The slab length of 19 feet should bring about an ultimate shrinkage of .12 inch per slab length, or .06 inch for each joint, and in general the cracks are actually that amount. The rods running through the rib head, having only about 30 inches assumed center to center of bond applications, will pass their elastic limit at about .04 inch total

stretch, or about .02 inch per joint. However, this amount of shrinkage represents only about one-third of the slab's ultimate. This amount of joint opening would produce in the steel passing through it a tension of about 40K* per square inch, or about 29K per square foot of slab cross section. If the rods are holding the slab against any further shrinkage, the slab tension would be about 126K per square foot of slab cross section. The rods, therefore, were sure to stretch, produce the cracks which have been seen, thus relieving all portions of the slab between the ribs from any crack menace.

Another crack between the toe block and the slab has developed for practically the full length of the structure. At this point, of course, occurs an abrupt change in section, and, of course, a construction joint. This crack continues with uniform thickness even when the top of the toe block is not more than 18 inches above the ledge. It indicates that the shrinkage of the slab about its center line, and not any shrinkage of the toe block, is the cause, and that the slab probably is held in position by its contact with the ribs.

The toe block shows cracks at each transverse construction joint, and, similar to the spillway, they run through to the ledge.

INTAKE STRUCTURE

The intake structure (Fig. 4) has an interesting history from the standpoint of assembly as affected by probable shrinkage. It occupies the river bed, and its construction required, first, a cofferdam so that the ledge might be exposed. This was built of dimension timber with checks filled with rocks. It was desired, however, to establish at an early date a second line of defence and of more permanent construction, so that immediately after the subsidence of any flood work could be started behind the coffer. The section which was built of concrete for this purpose is shown as No. 1.

This was designed to stand up with 6 feet of water over the top. It was an unwelcome part of the structure, since its top, and particularly its downstream faces, suggested internal water pressures of an undesirable direction. Since the resultant pressure diagram, under the complete section, was slightly trapezoidal, even in addition to the triangle of uplift water pressure, this detached chunk of concrete in stage No. 1 could not affect the stability of the structure, provided water could be kept out of the inclined back. The top and the intermediate planes were cleaned as thoroughly as possible and each provided with a copper baffle.

* K = 1,000 pounds

The stepped back was of interest, as it increased the length of the travel of any water which might get in, and it served to break the fall of the water when the section acted as a weir. Furthermore, this inclined back would reduce the severity of any local stresses in the pour next above, in the vicinity of the downstream edge of the top. These stresses would result from the upstream part of the upper pour resting on well-cured concrete, while the downstream part would be carried by much newer concrete, subject to more settlement.

With the penstocks on 45-foot centers, it was better to establish bulkheads on 45-foot centers, and in order to prevent splitting the 20-foot buttresses into two 10-foot compression legs, these bulkheads were located under the penstock center lines. The constant cross section of this chunky piece provided the best chances for crackless blocks, and, in addition, this section rapidly became enclosed by subsequent pours and sealed against losses of heat and moisture, and, of course, ultimately its upstream face became permanently saturated by the pond. At this time there was the experience with the 40 to 45 foot pours in the east wall which had no intermediate cracks.

Having thoroughly cleared the rock downstream from No. 1, the next section poured was section No. 2, also in 45-foot lengths, and then No. 3, the latter, of course, taking its dimension of 20 feet from the buttress.

The next section poured was portion No. 4 (T head in shape), with its head 45 feet between bulkheads and its stem 20 feet between forms. Upon the completion of this pour the concrete became flush with the top of the concrete in No. 1 cofferdam, and pouring was halted to permit the new concrete to get as much settlement behind it as possible, consistent with the program, before making the next pour. Thus it was provided that for portion 4A, the bottom areas would be as near alike as possible from the standpoint of future settlement. If the downstream portion of the bed under No. 4A were to settle more than the upstream portion, the No. 4A stage would be subjected to an intense upward pressure from No. 1, which would create an increased compression near the downstream faces of the buttresses. Nothing to date has indicated any disturbances arising from these dissimilar footings.

The bulkheads, one every 45 feet, centered under the penstocks, stopped upon intersecting the plane which forms the floor of the penstock trench, and also upon intersecting the top plane of No. 4A. It was intended that stage No. 5, 10 feet thick, 27 feet long, should seal the top of the bulkhead crack. This stage was not reinforced, and the bulkhead crack has continued up through No. 5 in two out of the four cases.

The concrete work continued upwards, except for the areas to be occupied by the penstocks, and this construction took the form of huge towers 20 feet thick, which ultimately became the buttresses of the finished structure. In order to complete the freight route to the shaft in the transformer service building at an early date, concrete arches were sprung between the buttresses so as to complete the supports for the upper side of the transformer bridge. Benches were recessed into the buttresses to support these arches. The latter were poured in the winter without keys, and these were therefore reinforced to counteract shrinkage. They were well protected with heavy canvas and heated by steam lines. Forms were built to carry the fluid dead weight of the first lift, followed by a wait of ten days, so that this first lift might set up into a structure. The reinforcing served another purpose in assisting the first lift to act as a base for the support of the fluid weight of the next.

FILLER BLOCKS FOR PENSTOCKS

From fifty to eighty days after the completion of the buttresses the "filler blocks" were sprung in between the buttresses bridging the gap left for the penstocks and their concrete collars. These filler blocks spanned 27 feet, which is the distance between recessed faces of buttresses. They were to be supported by lugs built monolithic with the blocks. Each block pour was bulky, and its shrinkage due to heat losses would be slow, but shrinkage due to decrease of free moisture was sure to be rapid because of the early demands which the setting concrete would make upon that free moisture. Such shrinkage might seriously impair the capacity of the lug's shear planes unless provision was made for it. The lugs provided the blocks with their only supports until the penstock collar should be poured. Even this might not furnish a support, since its tendency would be to shrink away from the under side of the filler block.

The general scheme for constructing these blocks was as follows: First there was built a very heavy "V" shaped platform of structural steel and dimension timber. The form floor was suspended from the steel so that stage No. 7 would enclose the steel and the latter would become part of the structure upon the setting up of the concrete.

In order to reduce the weight of the forms, portion No. 7 was split into upper and lower lifts and poured as follows: The lower half was poured, the fluid weight of which was carried by the forms. After three days the fluid dead weight of the upper half would be carried into the forms through the lower half with the latter acting as a solid instead of a fluid. After another wait of six days, during which the two lifts set up

into one structure, there was established the base upon which it was intended to build the remaining filler block lifts. The six-day waits between lifts were continued for reasons given below.

As has been indicated, the filler block was dependent upon its lugs, and it was necessary to protect them against any reduction in the capacity of their vertical shear planes. By delaying the weight of the next pour above, the shear plane was kept free from any downward movement of the filler block in relation to the lug, or any other disturbance, until the lift was well set up or cured. In order that the setting-up vertical shrinkage of the lift itself might produce a minimum movement or unsettling effect on the shear plane, the floor of the pour was placed as near the foot of the shear plane as possible.

There still was the effect that the tension, due to the horizontal shrinkage (which might ultimately reach $3/16$ inch), of the 27-foot length of pour might have upon the shear capacity of the lug plane. This proposition was attacked in two ways: first, by introducing reinforcing to the extent of .5 per cent, which was about as much as would permit well rodded concrete, thus transferring a part, at least, of the tension from the concrete to the steel. Next, the endeavor was made to reduce the tension by decreasing the friction of the lug on the floor of its recess, first, by sloping the floor, and next, by greasing with asphalt and two-ply felt.

Ultimately, if the lugs held the filler block against shrinkage, there would be a tension of about 1.4K per square inch in the cross section of the filler block, or about 2.8K per square inch tension through the lug shear plane. It was manifestly impossible to reinforce for this. Then it was decided to reduce the resistance to shrinkage as much as possible, in accordance with the sketch. The load per 5-foot filler block pour amounted to 425K, and since during setting-up there is a time when a given set of lugs carries the weight of two pours, there was developed a shear of 58 pounds per square inch on a lug shear plane.

During this setting-up there would be a total shear of 425K per set of lugs. A horizontal pull of 850K would turn the reaction on the recess floor from a vertical force into one parallel to the recess floor, and with the greased plane this appeared to be the maximum tension which could be produced by shrinkage. This tension, acting through the lug shear plane, would produce about 116 pounds per square inch. With the assistance of reinforcing steel it appeared that adequate protection had been provided for the lug shear plane against the effect of tension.

The concrete had to meet all of these shears and tensions when relatively green. This item led to conservative design and construction.

All of these precautions may seem unnecessary because no one really

believed that any one block would fall; it would soon jam. It might, however, have dropped enough to shear the copper seals and disturb the planes between the filler blocks and the buttresses enough to produce leaky joints. Since these joints extend through the structure we would not only have had leaks, but ultimately disintegration from frost. In order to protect these joints against leakage, due to opening of joints due to shrinkage, each joint was provided with a shaft and copper folded seals. Ultimately these joints were grouted, and now the whole scheme is successful in that the joints are tight.

The question naturally comes up as to why it was decided to carry the individual filler block lifts each by their own set of lugs rather than by building them on a concrete base over the penstocks, this base having adequate strength for spanning the gap, and also provided with benches projecting from the buttress faces for adequate support. The essence of this is that, in our analysis of the intake structure, the buttresses resist the entire water pressure. In other words, a 20-foot buttress has to take the horizontal force accumulated in a 45-foot length, and it becomes necessary to insure that the weight of the filler block was also fed into the buttresses as counterweight. If the whole weight of the filler block had been carried into the buttresses on a shelf just above the level of the top of the penstock trench, the portion of the buttress above that level would have been wholly without the beneficial effect of the filler block counterweight. The scheme which was used insured that for any horizontal plane in the buttress there was a counterweight 45 feet wide to match 45 feet of horizontal pressure above that plane. The stability of the structure, therefore, required the lug scheme which was used.

As the filler block began to overlap the arch, there was inserted a construction plane so that the former in shrinking and settling would not produce a drag upon the arch.

Other structures of concrete on this job required careful consideration of shrinkage. The above have been selected as offering good examples of how the shrinkage question enters into the design of concrete.

THE TYNGSBOROUGH PROJECT

BY A. E. KLEINERT *

THERE is now under construction at Tyngsborough, Massachusetts, a project which will replace the obsolete bridge over the Merrimack River and tracks of the Boston & Maine Railroad, and which will eliminate a dangerous curve on the interstate highway, route No. 3, at the westerly approach to the bridge.

ORIGINAL BRIDGE

The present bridge, consisting of four spans, was built during the period from 1872 to 1874, prior to which time a small ferry boat served those who wished to cross the river at this point. During the construction of this old bridge there were encountered difficulties arising from floods which followed severe storms. Just after the first span was completed a heavy rain set in, causing the river to rise. The logs, lumber and débris, which collected on the upstream side of the falsework, finally built up sufficient pressure to carry away all of the falsework for the bridge. A few months later, in December, when the second span was well along toward completion, a heavy, warm rain broke up a six-inch layer of ice on the river, produced another high stage, and wrecked the falsework again. This time, however, the span was carried downstream as far as Tyng's Island, about a mile away, before it sank. Later it was taken from the bottom of the river, sent back to the shop for repairs, and again placed in its position on the piers. The bridge was finally opened to the public in the spring of 1874. Since this date it has given excellent service, and is now being replaced only because it is unable to carry the loads demanded by the present-day trucking.

CONDITIONS TO BE MET

The preliminary studies made for the new structures indicated that locating the new bridge just upstream from the old bridge would give the best alignment to meet the changes made necessary in the road to eliminate the existing undesirable reverse curve. It also saves the cost of a temporary bridge for crossing the river during the period of construction,

* Assistant Structural Engineer, Massachusetts Department of Public Works.



FIG. 1. — LOCATION OF BRIDGE ACROSS MERRIMACK RIVER AT TYNSBOROUGH

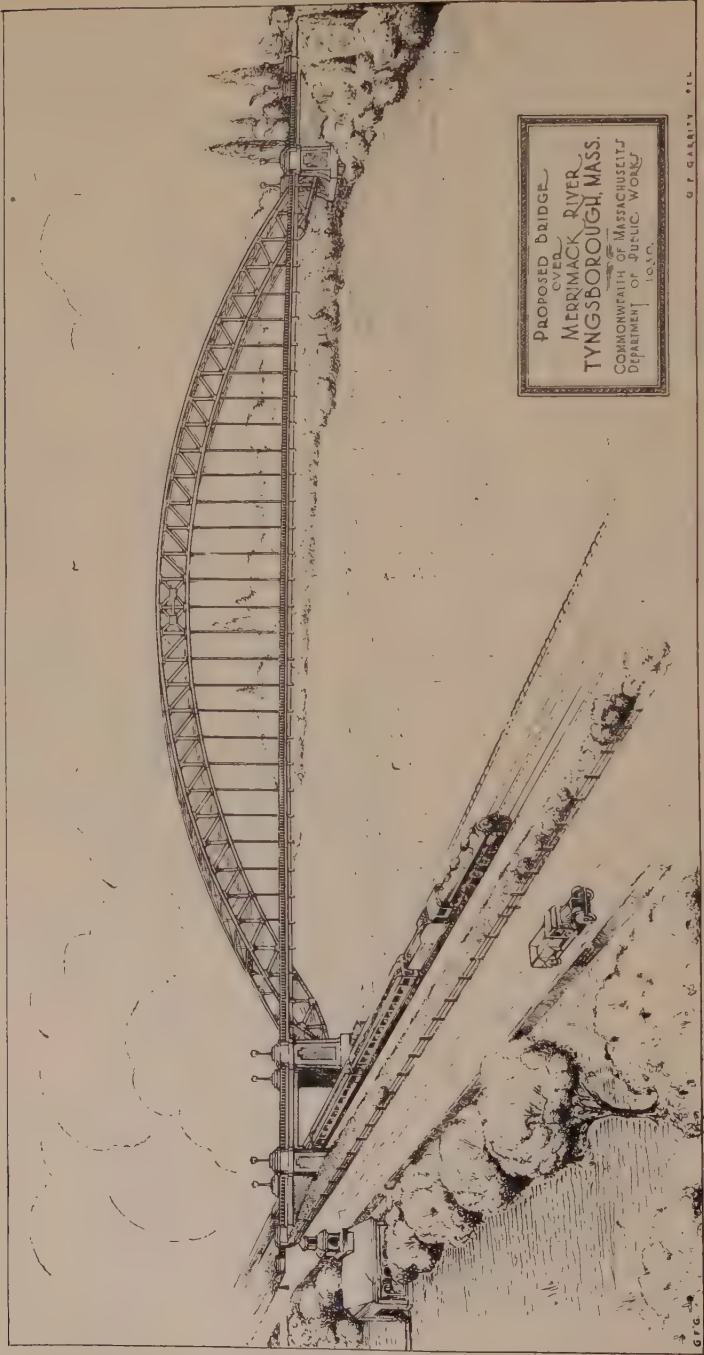


FIG. 2. — NEW TYNGSBOROUGH BRIDGE

Since the existing bridges will not be removed until the new bridges are completed. The types of bridges given consideration were those most adaptable to the conditions at the site. The bridge over the river will be a single span, through-arch truss, with the floor suspended from the trusses. The foundations favored an arch, since the borings indicated ledge about 10 feet below the surface of the ground on the west bank, and coarse sand and gravel with a hard firm underlying strata about 30 feet below the surface on the east bank. For the west abutment, then, the foundation will be carried directly to the ledge, and on the east the abutment will be supported on piles driven into the hard strata. A single span is advisable, in view of the river's record during flood stages. The most important factor, however, is the grade of the roadway on the bridge itself. It is necessary to have a difference of at least 10 feet between the elevations on the ends of the bridge, and a height of roadway at least 50 feet above the river bottom.

DESIGN FOR SPAN OVER THE RIVER

The design as finally adopted provides for a steel through braced arch with two truss ribs of 34 panels, each of 16 feet 1 inch in length, totaling 546 feet 10 inches between the centers of the end pins, a rise of 16 feet, and a depth of truss of 20 feet between working lines. For the dead load, the arch is three-hinged, but for all other loads it is two-hinged. To carry out this part of the design, the splice plates, which connect the halves of the trusses, will be riveted up after all of the dead load has been placed on the bridge, and at a time when the temperature is approximately 60 degrees Fahrenheit. The top and bottom chords are parallel and parabolic in shape. By use of this curve, the web stresses are reduced to a minimum, especially those for the dead load, which is practically a uniform load, with additional concentrations at the portals and the center panels. The trusses are spaced 34 feet on centers to provide for a 30-foot roadway between curbs which are 10 inches high. One 5-foot sidewalk is supported upon brackets cantilevered out from the ends of the floor beams on the downstream side of the bridge. Between panel points Nos. 4 and 31, the floor is suspended from the trusses by hangers. At each end of these hangers there is a $3\frac{1}{2}$ -inch pin connecting it to the floor beam at the lower end, and to the truss at the upper end. These pins provide for the differences in expansion between the floor and the trusses, — a difference which must be taken care of, because the truss expands from the ends toward the center, and the floor from the center toward the ends. At panel points Nos. 4 and 31 there are located box

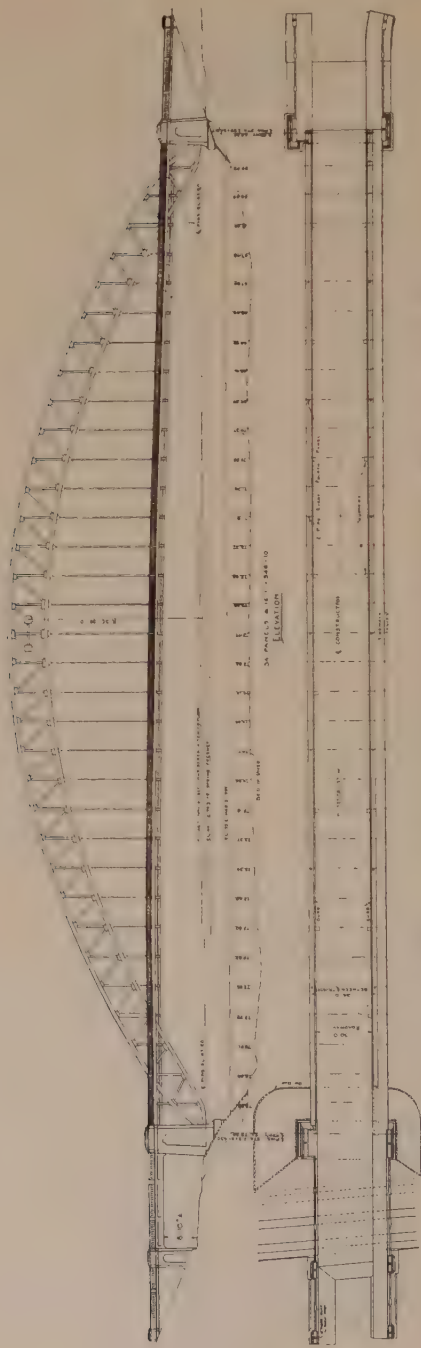


FIG. 3. — PLAN AND ELEVATION OF NEW TYNGSBOROUGH BRIDGE OVER MERRIMACK RIVER

portals which transfer the wind loads from the top chord lateral system to the bottom chord system. These portals also include expansion joints between the floor and the trusses, — joints which are so arranged that expansion may occur without interfering with the transfer of the wind loads from the floor lateral system to the trusses. From the portals all lateral loads are carried along the bottom chord bracing system to the abutments.

LOADINGS

The bridge has been designed for the class A loading according to the specifications of the American Association of State Highway Officials, and adopted by the American Railway Engineering Association March 12, 1929. Additional loadings are sidewalk, wind and temperature. The temperature loading is that resulting from a range varying by 60 degrees above and below 60°F. The secondary stresses are low in all but those members adjacent to the pins, and only at the center panels has it been necessary to add metal to provide for these stresses.

DETAILS

The sections used for the truss members are those found in standard practice. For the top chord there was designed the usual cover plate, web plates, and top and bottom angles. For the bottom chord a section consisting of two web plates and four angles was selected. Rolled I-beam sections proved to be the most economical for the web members, and are used. They are economical because the entire cross-sectional area may be considered to carry stress, and there is no extra weight for batten plates and lattice bars. A saving in shop work is made, for ordinarily only the ends of the beams are punched for field connections. Similarly, in other places in the bridge, rolled sections were found to be more economical than built-up sections, and were therefore used. The hangers, web members of the floor lateral system, floor beams, and, of course, the stringers, are all I-beam sections. In order to reduce the size of the gusset plates and splices, 1-inch rivets are used for the main field connections. All other rivets are 7/8 inch. During fabrication the trusses will be assembled in the shop at least four panels at a time, and the connections checked before the open holes are reamed for field riveting.

The fence is a new type recently developed by the Department. It is built up with two 2½-inch diameter wrought iron pipe rails, one top

and one bottom, with $\frac{3}{4}$ -inch wrought iron pales spaced 5 inches on center welded to them. It is expected that this fence will resist corrosion, which is a most troublesome failing of steel fences.

BRIDGE OVER RAILROAD

For the bridge over the railroad tracks there has been designed a steel tringer span with a concrete deck. Provisions have been made in the

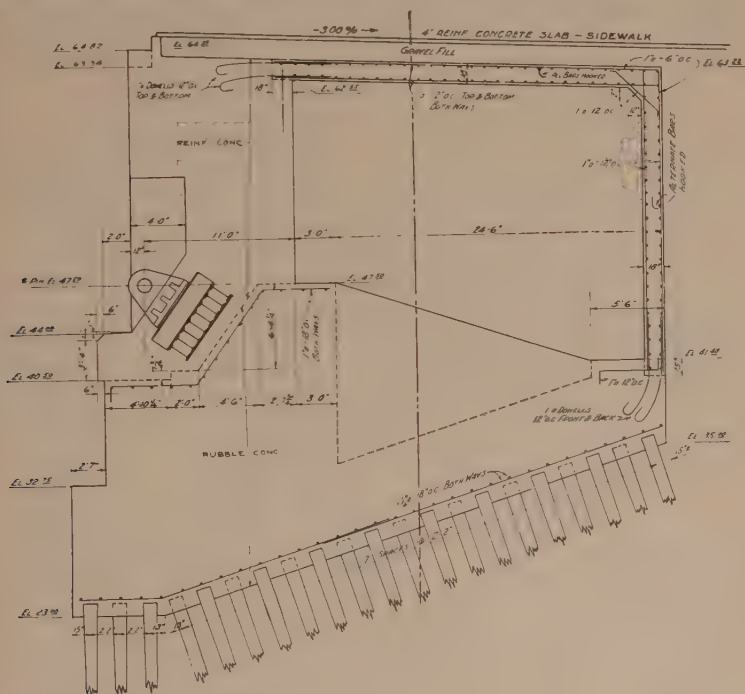


FIG. 6.—SECTION THROUGH EASTERLY PIER, SIDEWALK TRUSS

length of this structure to meet a future demand for a third and fourth track. As the elevation of the road surface on this bridge dominates the profile of both bridges and approaches, it is necessary to make the floor as shallow as possible. Consequently for the bridge over the tracks a 36-inch rolled I-beam section is used for the stringers, which with a concrete deck gives a minimum depth of floor, and at the same time proves to be economical. With this type of bridge no falsework or posting is required underneath during construction, and therefore there is little if any interference with the railroad traffic. To protect the steel from the corrosive

action of the locomotive exhaust, a 3-inch concrete covering will be poured around each stringer. The fence on this bridge will be of concrete to harmonize with the architecture of the concrete piers and abutments.

ARCH CULVERT

The elimination of the curve on the highway at the westerly end of the bridge requires the placing of a 40-foot fill over a reinforced concrete arch culvert which is to be 18 feet wide, 11 feet high, and 220 feet long.

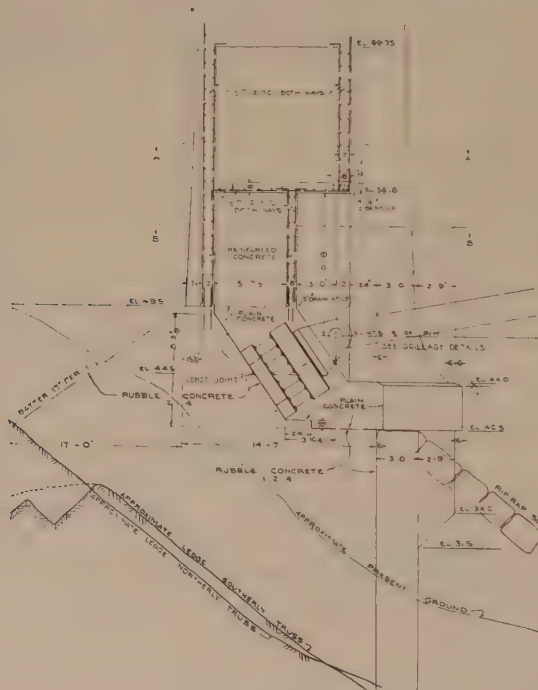


FIG. 7.—SECTION THROUGH WESTERLY PIER,
TYNGSBOROUGH BRIDGE

It is also necessary to make a 20-foot extension to the width of the existing reinforced concrete bridge on the highway approaches to the bridge in the town of Tyngsborough. And finally there is the customary work of excavation, filling in and surfacing for the road, and also the accessories, such as curbing and guard rail, all of which are necessary to correctly construct this project.

QUANTITIES

Some of the items and the quantities as estimated for the contract are as follows:

Excavation (cubic yards)	9,785
Rubble concrete (cubic yards)	4,179
Reinforced concrete (cubic yards)	2,409
Borrow (cubic yards)	55,650
Structural steel (pounds)	2,650,000
Reinforcing steel (pounds)	444,700
Piles (linear feet)	15,000

This project, the estimated cost of which is \$389,000, has been designed and is being executed by the Massachusetts Department of Public Works, of which Mr. Frank E. Lyman is Commissioner and R. K. Hale and H. A. MacDonald, Associate Commissioners. The engineering work is being done under the direction of Mr. Arthur W. Dean, Chief Engineer, Mr. G. E. Harkness, Bridge Engineer, Mr. F. C. Pillsbury, Project Engineer, and R. W. Coburn, Construction Engineer. The general contractor on this work is Simpson Bros. Corp. of Boston, and the subcontractor for the steel work is The Boston Bridge Works of Cambridge.

OF GENERAL INTEREST

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETINGS

Boston Society of Civil Engineers

JANUARY 28, 1931. — A joint meeting of the Boston Society of Civil Engineers and of the Boston Section, American Welding Society, was held this evening in Chipman Hall, Tremont Temple, and was called to order by President Lewis E. Moore at 7 P.M. About 225 members and guests were present. The meeting was preceded by a buffet supper, with an attendance of 140 members and guests.

The minutes of the previous meetings (November 19 and December 17, 1930) were declared approved as printed in the January JOURNAL.

The President announced the death of Charles A. Mason on December 31, 1930, a member since June 21, 1893, and of John E. Titus on January 18, 1931, a member since February 21, 1900.

The Secretary announced that the following new members were elected on January 28, 1931, by the Board of Government:

Grade of Junior: Alfred C. Anderson, Millard H. Aubey, William A. Bagloe, Charles E. Bateman, Roger S. Brown, William H. Burton, Jr., John G. Carlson, Joseph H. Coffey, Eugene B. Condon, Algott O. Forsberg, Kenneth H. Gilson, David E. Koff, Francis H. Ledyard, George K. Norton, Edward J. Parsons, Jr., Roger V. Patten, Robert W. Perry,

Charles V. Scher, Merton C. Smith, Theron B. Thompson, Albert P. Whitaker.

The President called upon Col. Frank M. Gunby, Past President, who made an announcement regarding the dinner of the Engineering Societies which will be held on February 11, 1931.

The President, after announcing that this meeting had been arranged jointly with the American Welding Society, called upon Mr. Harold P. Peabody, Chairman of the Boston Section of the American Welding Society, who commented briefly on the meeting and the subject to be presented. Mr. S. P. Waldron arranged the program and obtained the speakers for this meeting, and Mr. Philip N. Rugg, of the Welding Society, assisted the Social Activities Committee.

The subject of the technical program was "The Planning, Specifying, Super-vising and Welding of the New Fourteen Story Edison Building." Various phases of these subjects were presented by the following speakers: Mr. J. T. Whitney, President of the Whitney Engineering Company of Boston; Mr. J. L. Faden, Industrial Heating Engineer with the Edison Electric Illuminating Company of Boston; and Mr. A. G. Leake, President, Leake & Nelson Company, Bridgeport, Conn. These talks were illustrated by stereopticon slides and by motion pictures.

The meeting adjourned at 9.30 P.M.

EVERETT N. HUTCHINS, *Secretary*.

Designers Section

FEBRUARY 12, 1931. — A regular meeting of the Designers Section was called to order by the Chairman, Norman P. Randlett, at 6.10 P.M. in the Affiliation Rooms.

The reading of the minutes of the January meeting was omitted.

It was voted that the Chairman appoint a committee to make nominations for officers to be elected at the annual meeting of the Section in March. The Chairman appointed the following committee:

Waldo F. Pike, *Chairman*, Scott Keith and William D. Henderson.

The Chairman then introduced Mr. Raymond A. Freeman, who spoke on the subject of the paper written by Mr. A. S. Gelman and himself for the December, 1930, number of the JOURNAL, "The Angular Rotation Method for the Analysis of Multiple Span Frames." Mr. Freeman derived the fundamental equations which are used in this method, and showed how they would be applied to the solution of a typical problem, and explained the advantages of this method.

Mr. Freeman's talk was discussed by Prof. Frederic N. Weaver of Tufts College and Prof. Albert Haertlein of Harvard

Engineering School. General discussion then followed.

There were thirty-seven members and guests present.

The meeting was adjourned at 7.30 P.M.

LAWRENCE G. ROPES, *Clerk*.

Sanitary Section

FEBRUARY 4, 1931. — At 6 P.M., 15 members dined at Patten's Restaurant. At 7.30 P.M. the meeting was called to order by the Chairman, Harrison P. Eddy, Jr.

The Executive Committee was instructed to appoint a Nominating Committee to nominate officers for the following year. The Executive Committee appointed the following Nominating Committee: Prof. Gordon M. Fair, Chairman, Ralph W. Horne, F. W. Gilcreas.

The speaker of the evening was E. Sherman Chase of Metcalf & Eddy, who talked on "The Trials and Tribulations of a Sanitary Engineer Witness."

The paper was discussed by Messrs. Fair, Wright, Kingsbury, Fales, Coburn and Wells.

Twenty-two members and guests attended the meeting.

The meeting adjourned at 8.40 P.M.

GEORGE G. BOGREN, *Clerk*.

BOSTON SOCIETY OF CIVIL ENGINEERS

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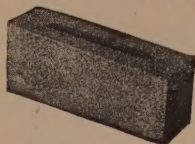
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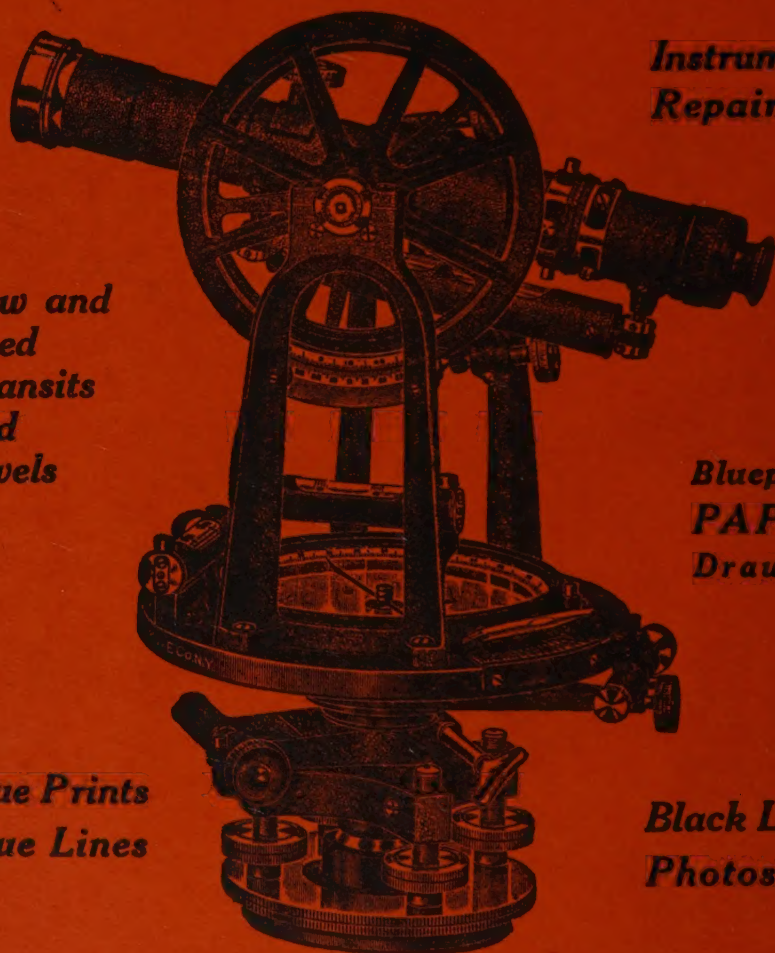
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